Numerical investigation of the resistance of precast RC pinned beam-to-column connections under shear loading

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Abstract

In precast technology, the effective design and construction is related to the behaviour of the connections between the structural members in order to cater for all service, environmental and earthquake load conditions. Therefore, the design and detailing of the connections should be undertaken consistently and with awareness of the desired structural response. In the research presented herein an analytical expression is proposed for the prediction of the resistance of precast pinned connections under shear monotonic and cyclic loading. The proposed formula addresses the case where the failure of the connection occurs with simultaneous flexural failure of the dowel and compression failure of the concrete around the dowel, expected to occur either when (a) adequate concrete cover of the dowels is provided ($d > 6\ D$) or (b) adequate confining reinforcement (as defined in the article) is foreseen around the dowels in the case of small concrete covers ($d < 6\ D$). The expression is calibrated against available experimental data and numerical results derived from a nonlinear numerical investigation. Emphasis is given to identifying the effect of several parameters on the horizontal shear resistance of the connection such as: the number and diameter of the dowels; the strength of materials (concrete, grout, steel); the concrete cover of the dowels; the thickness of the elastomeric pad; the type of shear loading (monotonic or cyclic); the pre-existing axial stress in the dowels; and the rotation of the joint. In addition, recommendations for the design of precast pinned beam-to-column connections are given, especially when the connections are utilised in earthquake resistant structures.

Keywords

Precast; pinned beam-to-column connections; pure shear; cyclic and monotonic response; shear resistance; nonlinear numerical investigation

1. INTRODUCTION

In recent years, there has been an increase in the use of prefabricated / off-site construction techniques, including precast concrete. Precast elements, such as structural members (beams, columns and slabs), architectural cladding panels and/or stair flights are being extensively introduced to the precast building construction or even used to buildings which are primarily constructed in-situ. A shortage of site tradesmen, the need to eliminate uncertainty in the construction process caused by inclement weather conditions and the general requirement for fast, reliable and economic construction techniques are among the main drivers.

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The precast structural systems are composed of precast concrete elements that must be properly connected to ensure the structural integrity of the whole structure. Therefore, the role of the connections between the precast members, i.e. the type of the connections and their position into the structural system, is of crucial importance, since their resistance affects the response of the whole system, especially under seismic loading.

Recently, considerable research on prefabrication has been reported worldwide. However, most of this research has been focused on the behaviour of specific types of precast systems and of their connections used by the precast construction industry and is not related to the behaviour of pinned beam-to-column connections, especially under cyclic or seismic loading. Such pinned connections are used in low-rise structures mostly in south-west Europe. These connections are not much used in US, except as "gravity frames" with other moment resisting frames or shear walls adopted as primary seismic resisting systems. Pinned beam-to-column connections are designed to allow rotations, due to the: (a) flexibility; (b) lower cost; and (c) more favourable behaviour they provide, especially in the case of large spans and pretensioned interconnected members.

Considerable information on the design and behaviour of various types of precast connections is given in the recently published *fib* Bulletin 43 [1]; however, emphasis is given to the behaviour of the connections under monotonic loading, while their seismic response is not covered sufficiently. An investigation on pinned connections made of steel dowels has also been reported by Leong [2] while significant work related to the behaviour of several types of precast connections has been presented by many other researchers (Orlando *et al.* [3], Tanaka and Murakoshi [4]; Rahman *et al.* [5]; Joshi *et al.* [6] among others).

Recently, significant experimental and numerical research on the seismic behaviour of precast structures with pinned connections was carried out in the framework of two research projects of the European Commission: the “Growth” FP5 project “Precast EC8: Seismic behaviour of precast concrete structures with respect to Eurocode 8 (Co-Normative Research)”, which concluded in 2007, and its follow-up, the FP7 project “SAFECAST: Performance of innovative mechanical connections in precast building structures under seismic conditions”, which was completed in 2012. The first project focused on the overall behaviour of precast structures and on the global ductility that can be attained (Negro *et al*. [7], Carydis *et al.* [8]). However, a detailed investigation on the seismic response of the connections themselves was not performed. This investigation was performed within the second project (SAFECAST [9], [10]). Extended research on the seismic response of precast industrial buildings is also presented by several researchers (Fischinger *et al.* [11], Apostolska *et al.* [12]). A case study of an industrial building in Italy was used recently for the seismic performance of precast reinforced concrete buildings with pinned connections by Clementi *et al.* [13] and the seismic risk of precast industrial buildings with strong connections is commented by Kramar *et al.* [14].

With regard to the numerical modelling of connections in precast structures, recently, Zoubek *et al.* [15], [16] and Kremmyda *et al.* [17] presented numerical models of the pinned connections investigated experimentally in the framework of the SAFECAST project.

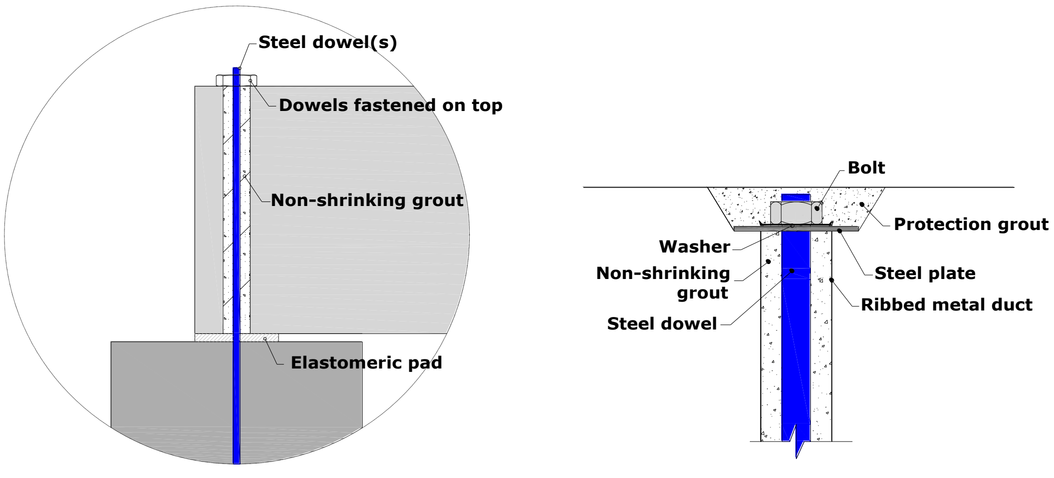
The aim of the present paper is to extend the experimental investigation undertaken within the SAFECAST project on precast RC pinned beam-to-column connections under monotonic and cyclic pure shear loading to a more rigorous numerical investigation on the effect of each parameter, including additional ones which were not examined experimentally. In particular, the following parameters were considered in this study: the number and diameter of the dowels; the strength of the materials (concrete, grout, steel); the concrete cover of the dowels; the thickness of the elastomeric pad; the type of loading (monotonic or cyclic); the pre-existing axial stress in the dowels; and the beam-column relative rotation at the joint.

Based on the numerical results, a refined expression for the estimation of the shear resistance of pinned connections in the case where the failure of the connection occurs with simultaneous flexural failure of the dowel and compression failure of the concrete around the dowel, is proposed for design purposes, which is consistent with the available experimental data from the SAFECAST project. The analytical investigation was undertaken by applying the nonlinear FE model proposed by Kremmyda *et al.*, which was developed with ABAQUS [18].

2. OVERVIEW OF A PINNED BEAM-TO-COLUMN CONNECTION

Typical precast pinned beam-to-column connections are made of one or two steel dowels (ribbed or threaded bars) which protrude from the top of the column or the upper face of column corbels in case of multi-storey buildings and insert into vertical sleeves foreseen at the beam's ends. The sleeves are filled with non-shrinking grout infill, while the dowels can be free or bolted at their top. It is recommended to fasten the dowels at the top of the beam in order to: (i) prevent beam overturning during erection (before grouting) due to an accident or an unexpected seismic event; and (ii) ensure the integrity of the connection during a strong earthquake. The beams are usually seated on elastomeric pads.

A typical pinned beam-to-column connection is shown in Fig. 1(a) while the detail of proper dowel fastening is given in Fig. 1(b).



(a) (b)

Fig. 1. (a) Detail of a typical precast pinned beam-to-column connection; (b) Detail of fastening at the top of the dowel.

3. SHEAR RESISTANCE OF PINNED CONNECTIONS: TYPES OF FAILURES AND EXISTING FORMULAE

The resistance of typical precast RC pinned beam-to-column connections, as described in Section 2, is provided by the steel dowels. For small shear deformations of the connections, the dowels are subjected mainly to shear loading (dowel action), while for large deformations the dowels are stressed in both shear and axial loading, as there is significant elongation of the bars due to the relative displacement of the beam with respect to the column and the rotation of the connection.

Failure of the connection occurs under three potential mechanisms (*fib* Bulletin 43 [1]): (i) steel shear failure; (ii) concrete splitting failure; (iii) exceedance of the dowels' flexural strength accompanied with simultaneous crushing of the surrounding concrete under high compressive stresses (Vintzeleou and Tassios [19], Psycharis and Mouzakis [20]). The type of mobilised failure mechanism depends on the strengths and dimensions of the steel dowels as well as the position of the dowels relative to the concrete element's boundaries. A weak dowel embedded in a strong concrete element might fail in shear of the dowel itself. In case of a strong steel dowel in a weak element or placed with small concrete cover, concrete splitting or steel flexural failure with simultaneous concrete crushing are more likely to develop.

However, when adequate concrete cover of the dowels is provided ($d > 6\ D$) and adequate confining reinforcement (as defined later in Section 3) is foreseen around the dowels in the case of small concrete covers ($d < 6\ D$), the third ductile failure mechanism as aforementioned is to be mobilised (Vintzeleou and Tassios [19]; Pauley *et al.* [21]; Zoubek *et al.* [22]).

For the case of adequate concrete cover of the dowels several empirical formulae have been proposed by various researchers for the calculation of the design (horizontal) shear resistance, R_d, of pinned connections, presented in the following. For the case of small concrete covers, less investigation has been carried out, with the most recent and notable ones being those by Psycharis and Mouzakis [18], Zoubek *et al.* [20].

Rasmussen [23] investigated experimentally the behaviour of one-sided dowels under eccentric monotonic shear loading applied at a distance e from the concrete edge and concluded that the design shear resistance of n dowels of diameter D is given by:

$$R_{d,m} = 1.30 \cdot n \cdot D^2 \cdot \left(\sqrt{1 + (1.3 \cdot \varepsilon)^2} - 1.3 \cdot \varepsilon \right) \cdot \sqrt{f_{cd} \cdot f_{yd}} \tag{1}$$

where f_{cd} and f_{yd} are the design strength of the concrete in compression and the design yield stress of the dowel, respectively, and

$$\varepsilon = 3 \cdot \frac{e}{D} \cdot \sqrt{\frac{f_{cd}}{f_{yd}}} \tag{2}$$

Eq. (1) is valid only if adequate concrete cover exists around the dowels, typically larger than 5 D in the direction of loading and 3 D in the transverse direction. For monotonic shear loading applied at the joint interface ($e = 0$), Eq. (1) becomes:

$$R_{d,m} = 1.30 \cdot n \cdot D^2 \cdot \sqrt{f_{cd} \cdot f_{yd}} \tag{3}$$

Vintzeleou and Tassios [19] proposed the following expressions, based on experimental and theoretical approaches and are valid only for concrete covers in the direction of the loading at least equal to 6 D:

For monotonic loading: $$R_{d,m} = 1.30 \cdot n \cdot D^2 \cdot \sqrt{f_{cd} \cdot f_{yd}} \tag{4}$$

For cyclic loading: $$R_{d,c} = 0.65 \cdot n \cdot D^2 \cdot \sqrt{f_{cd} \cdot f_{yd}} \tag{5}$$

It must be noted that these formulae were derived from experiments on double-sided dowels embedded in concrete without confining reinforcement around the dowels and for concrete blocks being practically in contact, without the gap of the elastomeric pad. Also they were calculated for relatively small displacements, before any strain hardening of the dowels occurred.

Psycharis and Mouzakis [20], using the experimental data obtained within the SAFECAST project, proposed the following expressions for the shear resistance of pinned connections under cyclic loading:

For $d/D > 6$: $$R_{d,c} = C_0 \cdot n \cdot D^2 \cdot \sqrt{f_{cd} \cdot f_{yd}} \tag{6}$$

For $4 \le d/D \le 6$: $$R_{d,c} = C_0 \cdot (0.25 \cdot d/D - 0.50) \cdot n \cdot D^2 \cdot \sqrt{f_{cd} \cdot f_{yd}} \tag{7}$$

in which d is the concrete cover of the dowels in the direction of loading and C_0 is a correction factor to account for the reduction of the strength due to the rotation that takes place at the joint. Concrete cover with thickness $d < 4D$ should be avoided. The coefficient C_0 varies from 0.90 to 1.10 depending on the magnitude of the expected joint rotations: for flexible columns, for which large joint rotations may occur, a value of C_0 around 0.90 to 0.95 is suggested; for stiff columns and walls, for which small joint rotations are expected, this coefficient can be increased. The maximum value of C_0 is 1.10 for practically zero joint rotations. For design purposes, a safety factor γ_R should be considered in conjunction with the above formulae, which typically varies from 1.20 to 1.30. The above-mentioned empirical formulae were derived from experimental results and, thus, they are valid for the specific conditions under which the tests were performed. Since the number of the experiments was limited, many parameters were not investigated in depth, or were not investigated at all.

Zoubek *et al.* [22] provided explicit experimental and numerical investigation of the behaviour of pinned connections with relatively small concrete cover of the dowels. The role of the confining reinforcement around the dowels in such cases was thoroughly investigated and a new procedure for the estimation of the resistance against splitting failure was proposed. Taking into account an appropriate strut and tie model of the connections, the effect of stirrups on the resistance of the connection and the type of failure was considered. When there are no stirrups in the critical region around the dowel, the failure is brittle. It occurs when the principal tensile stresses exceed the tensile strength of the concrete. However, usually, stirrups change the type of failure to ductile with a considerable effect on the strength of the connection.

Considering that one or more layers of stirrups (with any configuration) are provided in a critical region, h_{crit}, around the dowel where concrete rupture is typically observed, Zoubek *et al.* concluded that the strength of the dowel connection (under ductile failure), R_d, is defined as the force applied to the dowel(s) when the first layer of stirrups yields, see Eqs. (7)-(9):

$$h_{crit} = 2.5 \cdot D + c - a \tag{7}$$

$$n_s = h_{crit} / s + 1 \tag{8}$$

$$R_d = n_s \cdot A_{s1} \cdot f_{yd} \tag{9}$$

in which n_s is the number of engaged stirrups, c is the distance of the dowel to the axes of the stirrups and α is the vertical distance of the first layer of stirrups from the top of the column. However even if the resistance of the dowel connection is sufficient by the aforementioned formula, the resistance of the connection should be always also considered for local failure of the surrounding concrete under compression with simultaneous yielding of the steel dowel [22].

Utilising the research conducted by Zoubek *et al.* and by using collected experimental and numerical results, the authors derived an enhanced expression for the estimation of the shear resistance of pinned connections failing under combined steel/concrete failure, in which all the main parameters are included. The formula is valid for the cases of adequate concrete cover of the dowels is provided ($d > 6\ D$) and the cases of small concrete covers ($d < 6\ D$) with confining reinforcement around the dowels (in a critical region, h_{crit}, as defined by Zoubek *et al.*), capable of undertaking in tension the expected shear force applied to the connection without yielding.

4. EXPERIMENTAL DATA USED

The experimental data used to calibrate the formula that is proposed for the estimation of the shear resistance of pinned connections were obtained within the SAFECAST project [9]. Detailed information on the experimental investigation is given in Psycharis and Mouzakis [20]. The experimental campaign included a series of monotonic and cyclic tests on specimens that simulated an isolated pinned beam-to-column connection made of steel dowels (Fig. 1). The specimens were composed of two precast parts that simulated the end parts of a beam and a column connected by one or two steel dowels (Fig. 2). The dowels were bolted at their top and the gap around the dowels at the beam end was filled with non-shrinking grout. The beam was seated on an elastomeric pad of 2 cm thickness.

In total 22 tests were performed. Each specimen was subjected to monotonic (in pull or push direction) or cyclic displacement-controlled loading, applied at the rear end of the beam. The driving force was applied exactly at the level of the joint through a special device which allowed only uniaxial application of the loading, in order to achieve pure shear conditions without rotations. For the cyclic loading, three cycles were performed at each displacement amplitude.

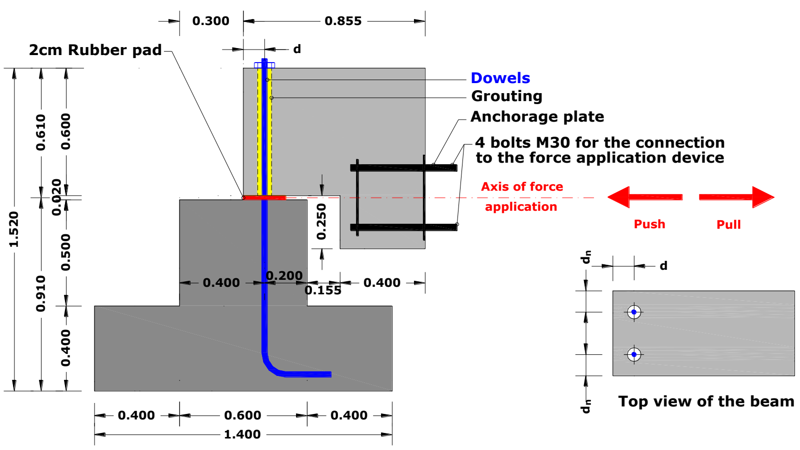


Fig. 2. Layout of the specimens used in the SAFECAST project.

The following parameters were investigated: the diameter of the dowels; the number of dowels; the concrete cover of the dowels in the loading direction; and the strength of the grout infill. The reinforcement of all specimens was the same and included 5 hoops ∅12/50 (hoops of 12 mm diameter spaced at 50 mm from centre-to-centre) at the lower 0.30 m of the beam and 3 hoops ∅12/100 at the remaining 0.30 m of the beam height.

5. SHORT DESCRIPTION OF THE NUMERICAL MODEL

The numerical model simulated the specimens' layout of Fig. 2 using 3D solid continuum elements within the environment of ABAQUS [18]. The model was an exact representation of the test specimens and was composed of five parts: (a) the interconnected concrete parts (beam and column); (b) the steel dowels; (c) the elastomeric pad and (d) the steel plate provided for the application of the imposed loading. A different material than the concrete of the beam and column was assigned to the grout around the steel dowels. The reinforcement of the specimens was not explicitly included in the model in order to facilitate the analysis process and its contribution was taken into account by the characteristics of confined concrete. Specifically, the Chang & Mander [24] stress-strain relationship for confined concrete was used. A numerical investigation about the effect of various configurations of confining reinforcement around the dowels on the cyclic capacity of beam-to-column connections was undertaken by Zoubek *et al.* [22]. The nonlinear behaviour of the concrete and the grout was modelled using the Smeared Cracking Model of ABAQUS. Tension stiffening was accounted by applying a fracture energy cracking criterion specified by a relevant stress-displacement response which required the definition of a characteristic crack length.

For the modelling of the steel dowels, the classic Plastic Model was used with stress-strain relationship according to the experimental data. The elastomeric pad was considered to behave elastically. Elastic behaviour was also assigned to the steel plate that was provided at the free end of the beam, which was used for the application of the driving force. Detailed information on the model is given in Kremmyda *et al.* [17].

The numerical model was calibrated and validated against the experimental data and was proved capable to predict satisfactorily the response of pinned connections under both monotonic and cyclic loading. A discrepancy was observed only in the case of small concrete cover around the dowels ($d < 6D$) under large imposed displacements. However the yielding point and/or the maximum values (which are of interest within the present numerical investigation) were satisfactorily predicted for all tests [17].

6. PARAMETRIC INVESTIGATION OF THE SHEAR RESISTANCE

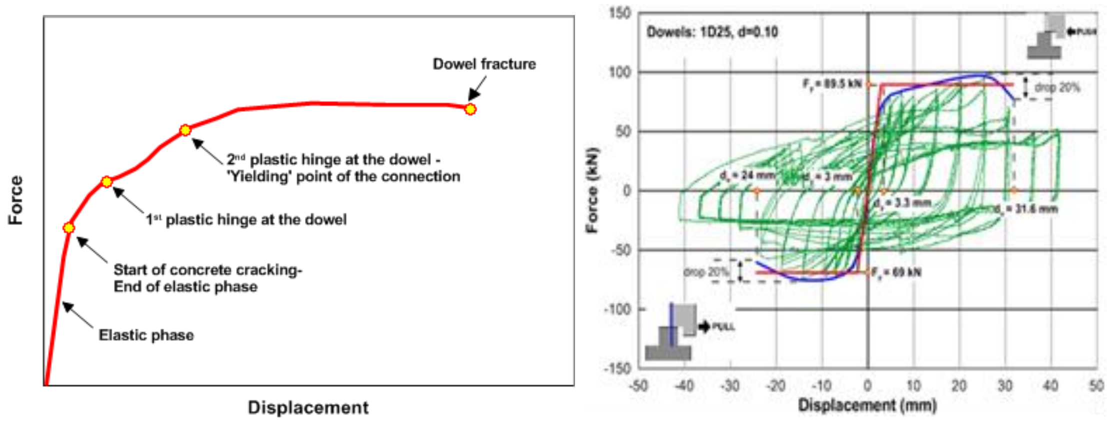
The parameters that are investigated analytically herewith are: the number, n, and diameter, D, of the dowels; the materials' strength (concrete, grout, steel); the concrete cover of the dowels in the loading direction, d; the concrete cover of the dowels in the normal to the loading direction, d_n; the thickness, t, of the pad that is placed between beam and column; the effect of pre-existing axial stresses in the dowel; and the relative beam-column rotation at the joint.

From the aforementioned parameters, only the number and the cross section of the dowels and their cover were investigated with the experiments performed within SAFECAST, while the remaining parameters were kept constant in all experiments. Due to the lack of enough experimental data, the above-mentioned nonlinear FE model, properly calibrated, was utilised for the rigorous investigation of the effect of all the above-mentioned parameters on the shear resistance of pinned connections.

The parametric investigation presented in the ensuing concerns only cyclic loading, thus, the proposed formula can be directly applied for the design of pinned connections against seismic action. This is generally on the safety side, since shaking table experiments on precast frames with pinned connections under real earthquake excitations (Psycharis and Mouzakis [25]) have shown that the dynamic resistance of the connections is rather larger than the one predicted by the static cyclic tests.

A typical force-displacement envelope curve of the response of a pinned beam-to-column connection is given in Fig. 3a. Initially an elastic phase is observed before the start of concrete cracking. Afterwards the first plastic hinge at one side of the dowel (with regard to the joint interface) is developed and the connection continues to respond elastically with increasing strength but with reduced stiffness. After the formation of a second plastic hinge at the other side of the dowel ('yielding' point of the connection), the failure mechanism of simultaneous flexural failure of the dowels and compression failure of the concrete is mobilized up to the fracture of the steel dowel.

The seismic design of precast structures with pinned beam-column connections is based on the concept that the prevailing energy dissipation mechanism should be through plastic rotations within critical regions of the columns, while the connections remain in the elastic region. In Eurocode 8 [26] such connections are termed as 'overdesigned connections'. Therefore in the numerical results that are presented in the following, the ultimate shear resistance of the connection, R_u, was assigned to the minimum value of the 'yield' strengths achieved in the push and the pull direction. These 'yield' strengths were determined from the idealised elastic-perfectly plastic bilinear representation of the corresponding force-displacement back-bone diagram (see Fig. 3b). The idealized elastic-perfectly plastic bilinear respresentation of the force-displacement diagram was developed according to Section B.3, Annex B of Eurocode 8.



(a) (b)

Fig. 3. (a) Typical force-displacement envelope curve of the response of a pinned beam-to-column connection; (b) Idealised bilinear force-displacement diagram

6.1. Effect of the number and the diameter of the dowels

All the available experimental data show that the ultimate shear resistance of the connection, R_u, is proportional to the total cross section of the steel dowels, which, in

Eqs. (3)-(7) is expressed by the product ($n{\cdot}D^2$), where n is the number of dowels and D is their diameter.

This linear relationship between R_u and the area of the cross section of the dowels was also confirmed from the numerical investigation, as shown in Fig. 4, in which R_u is plotted versus $n{\cdot}D^2$ for connections with one or two dowels with diameter D ranging from 10 mm to 32 mm. Therefore one can write:

$$R_u \propto n \cdot D^2 \tag{8}$$

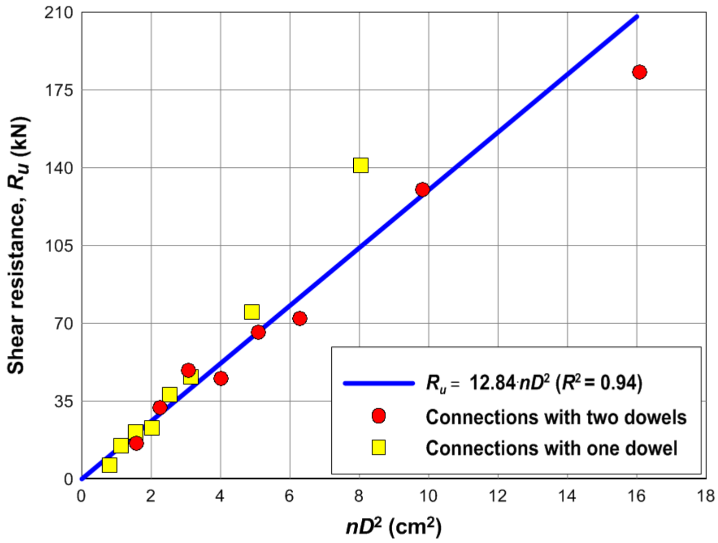


Fig. 4. Ultimate shear resistance, R_u, versus $n{\cdot}D^2$ for connections with one or two dowels and diameter D = 10 mm to 32 mm

6.2. Effect of the strength of the concrete and the grout

In all the existing numerical expressions, as the ones mentioned in Section 3 (Eqs. (1) and (3) to (7)), the effect of the concrete strength on the shear resistance is taken into account with the square root of its value. However, in practical applications in precast pinned beam-to-column connections, the sleeves for the insertion of the dowels are filled with non-shrinking grout which, in general, has different strength than the concrete of the beam or the column. Then, the question that arises is whether the strength of the grout or the strength of the concrete should be used in these equations for the estimation of the ultimate resistance of the connection. In *fib* Bulletin 43 [1] it is proposed that the larger strength among the concrete and the grout must be taken into account.

In order to check this assumption, two sets of parametric investigations were performed. In both cases, the connection was made of 2∅25 dowels with d = 0.10 m. In the first set of analyses, the compressive strength of the grout, $f_{c,g}$, was kept constant, equal to 23 MPa, while four values were assigned to the strength of the concrete, $f_{c,c}$, ranging from 25 MPa to 40 MPa, all being larger than $f_{c,g}$. In the second set, the strength of the concrete, $f_{c,c}$, was kept constant, equal to 35 MPa, and five values of the strength of the grout, $f_{c,g}$, were checked, two lower than $f_{c,c}$, namely 23 MPa and 30 MPa, and three larger than $f_{c,c}$, namely 40 MPa, 45 MPa and 50 MPa.

The results are presented in Table 1. It is seen that in the first case, in which $f_{c,c} > f_{c,g}$, the ultimate shear resistance increases as the larger strength, $f_{c,c}$, increases. In the latter case, in which $f_{c,c}$ was kept constant and $f_{c,g}$ was increasing, the ultimate shear resistance was not affected by the increase in $f_{c,g}$ as long as $f_{c,g} < f_{c,c}$ but, for $f_{c,g} > f_{c,c}$, it was increasing with $f_{c,g}$.

Table 1. Ultimate shear resistance for varying concrete and grout compressive strength.

Set of analysis	Grout strength $f_{c,g}$ (MPa)	Concrete strength $f_{c,c}$ (MPa)	Shear resistance R_u (kN)
1	23	25	98
	23	30	109
	23	35	128
	23	40	130
2	23	35	128
	30	35	129
	40	35	132
	45	35	135
	50	35	138

Normalized cumulative results are presented in Fig. 5a, in which R_u is plotted versus the maximum compressive strength of the two materials, $f_{c,\max} = \max(f_{c,c}, f_{c,g})$. A regression analysis of the numerical data showed that R_u is practically proportional to $\sqrt{f_{c,\max}}$, as suggested in *fib* Bulletin 43 [1]. Thus, one can write:

$$R_u \propto \sqrt{f_{c,\max}} \tag{9}$$

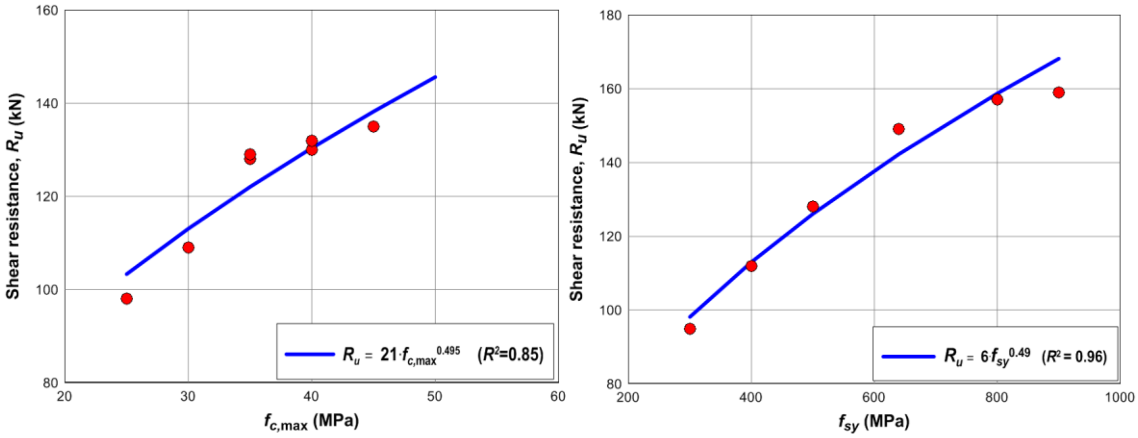


Fig. 5. (a) Variation of the ultimate resistance R_u with the maximum compressive strength between concrete and grout; (b) Variation of the ultimate shear resistance with the yield stress of the steel of the dowels.

6.3. Effect of the strength of the steel of the dowels

In all existing expressions (see Section 3), the effect of the strength of the steel of the dowels on the ultimate resistance of the connection is taken into account with the square root of the yield stress. To check the validity of this assumption, analyses were performed for five values of the yield stress of the dowels, f_{sy}, varying from 300 MPa to 900 MPa, while all other parameters were kept constant. The results are presented in Fig. 5b.

A regression analysis of the numerical data verified that R_u is practically proportional to $\sqrt{f_{sy}}$. Thus, one can write:

$$R_u \propto \sqrt{f_{sy}} \tag{10}$$

6.4. Effect of the concrete cover of the dowels

The experimental campaign performed within the SAFECAST project showed that the shear resistance of the connection is affected by the concrete cover of the dowels, if the dowels are placed close enough to the edges of the beam or the column. Small concrete cover in the direction of loading, lower than $6D$, leads to early spalling of the concrete while, in the case of thick covers, local failure of the concrete in compression occurs in the vicinity of the dowel [19].

In the present research, the concrete cover refers to the distance of the centre of the dowel from the concrete edge (Fig. 6a). The cover of the dowels in the loading direction is denoted by d and the cover in the normal to the loading direction by d_n. The performed investigation concerned values of d/D varying from 4 to 10 and values of d_n/D varying from 4 to 6. Values of concrete cover smaller than 4 D are not possible (and not recommended) taking into account the geometrical restrictions due to the usual arrangement of reinforcement and code-compliant concrete covers (see Fig. 6b [27] and [28]).

Concerning the effect of the cover d_n in the normal to the loading direction, the reduction of the shear resistance for $d_n/D = 4$ and $d_n/D = 5$, in comparison with the maximum resistance $R_{u,\max}$ obtained for $d_n/D = 6$ and $d/D = 10$, is depicted in Fig. 7 for several values of the cover in the loading direction, d/D. It is seen that the drop in resistance due to smaller cover d_n was less than 3% for $d_n/D = 4$, except of the case $d/D = 4$ when the strength dropped by 6.5%, and less than 2% for $d_n/D = 5$. It was concluded, therefore, that the effect of d_n is not important and d_n/D was not considered a parameter that affects the shear resistance, provided that a minimum value of $d_n/D \geq 4$ is guaranteed. This conclusion is in accordance with the results reported in [20].

Contrary to the cover d_n, the effect of the cover d in the loading direction is important. As expected from the experimental data, the numerical analyses showed that the response is not symmetric (different in the push and the pull direction, see Fig. 2) for low values of concrete cover ($d < 6D$) and that the shear resistance of the connection for the minimum concrete cover examined ($d = 4D$) is significantly smaller than the one for the larger ($d = 10D$). The results are presented in Fig. 7, in which the variation of the normalized shear resistance, $R_u/R_{u,\max}$, with the ratio d/D is shown, where $R_{u,\max}$ is the maximum resistance corresponding to quite large values of d/D, set equal to the calculated resistance for $d/D = 10$. It is seen that the strength drop decreases as d/D increases and $R_u/R_{u,\max}$ tends to become equal to unity.

In order to take under consideration the reduced resistance for small values of the concrete cover d, a reduction coefficient a_{cov} is introduced, thus one can write:

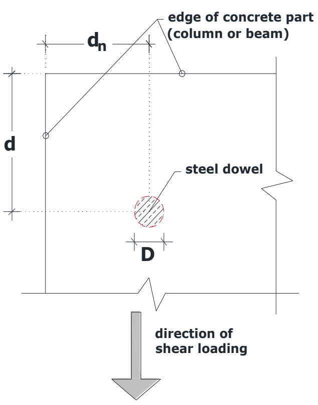
$$R_u = a_{cov} \cdot R_{u,max} \tag{11}$$

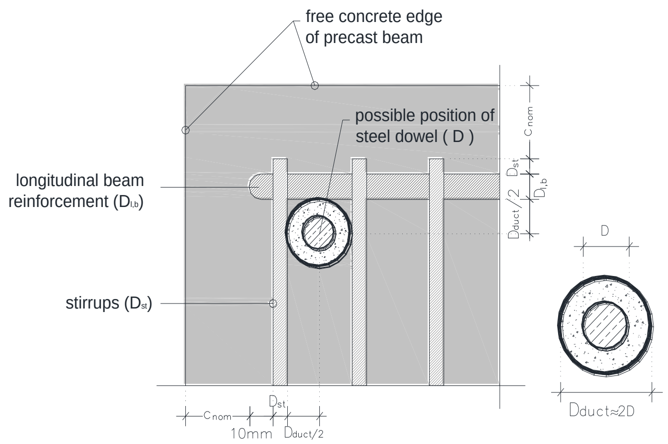
Performing a least square analysis on the numerical results, the following expression was derived for the calculation of the coefficient a_{cov}:

$$a_{cov} = -0.014 \cdot (d/D)^2 + 0.252 \cdot (d/D) - 0.134 \quad \text{for } d/D \leq 9 \tag{12a}$$

$$a_{cov} = 1.00 \quad \text{for } d/D > 9 \tag{12b}$$

Eq. (12) is depicted in Fig. 8 with a solid line and it is seen that it predicts quite satisfactorily the numerical results.





(a) (b)

Fig. 6. (a) Definition of concrete covers d and d_n with regard to the direction of loading; (b) Minimum practical distance of a dowel to the free edges of a concrete beam [25]

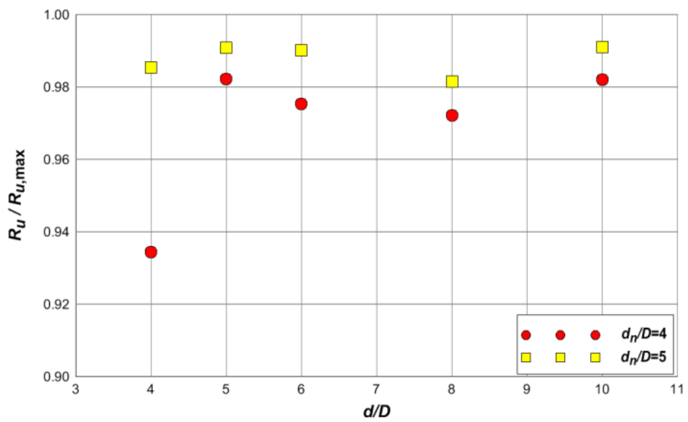


Fig. 7. Reduction of the shear resistance with reduced values of the cover d_n in the normal to the loading direction and varying values of the cover d in the direction of loading.

6.5. Effect of the thickness of the elastomeric pad

It is common in precast structures with pinned beam-to-column connections to use elastomeric (neoprene) pads for the proper sitting of the beams on the columns. Their purpose is to eliminate any anomalies of the contact surfaces, to ensure the even distribution of the stresses at the column face, to allow small displacements caused by thermal effects, prestress of the beam, creep, etc., and to prevent the impact between beam and column due to the rotation of the joint during the seismic response.

In practice, the thickness of the elastomeric pad, t, varies from 1 to 4 cm. In the expressions for the calculation of the shear resistance presented in section 3, only the

Rasmussen formula (Eq. (1)) considers the effect of the eccentricity e of the shear force with respect to the face of the joint, which could be used to simulate one half of the pad thickness. This formula, however, was derived from tests on one-sided dowels, where no axial forces were developed in the dowels, thus it cannot count for the complicated stress field developed in pinned beam-to-column connections.

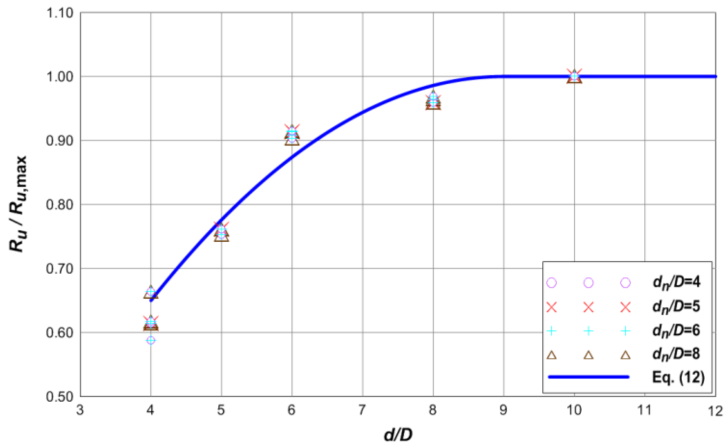


Fig. 8. Reduction of the shear resistance with the cover d in the loading direction.

In the case a precast pinned beam-to-column connections with an elastomeric pad for the sitting of the beam, the response mechanism seems to differ. The thickness of the elastomeric pad is related to the unbonded length of the dowel between the two interconnected elements and affects directly the axial stresses developed in the dowel under a given horizontal displacement of the beam. In the case of using a thicker elastomeric pad, under a given horizontal displacement of the beam, the inclination of the dowel is larger and greater axial (tensile) stresses are developed causing its plastic elongation and permanent increase of their length up to their breaking point.

In the numerical investigation, the effect of the pad thickness, t, on the shear resistance, R_u, of the connection was examined for eight values of t, ranging from 0.5 cm to 4 cm, and for connections made of 2Ø25 dowels with cover d = 0.10 m. The shear strength versus the pad thickness is presented in Fig. 9a, in which t is normalized with respect to the reference thickness t_0 = 2 cm (most commonly pad thickness used) and R_u is normalized with respect to the corresponding shear resistance, R_{u0}. It is seen that the ultimate shear resistance of the connection decreases as the pad thickness increases.

Based on the obtained results, it is suggested that a correction coefficient α_t should be used for the calculation of R_u when a pad of thickness different than 2 cm is applied, i.e.

$$R_u = a_t \cdot R_{u0} \tag{13}$$

Performing a regression analysis on the numerical results, the following equation was derived for the calculation of the coefficient a_t:

$$a_t = 1.25 - 0.25 \cdot t / t_0 \tag{14}$$

in which t the pad thickness in cm and t_0 = 2 cm.

Eq. (14) shows good agreement with the numerical results (the coefficient of determination is R^2 = 0.92), as evident from Fig. 9a, where Eq. (14) (blue continuous line) is plotted together with the Rasmussen formula (Eq. (1)) for e = 2t (green dashed line) for comparison. It is seen that the Rasmussen formula deviates considerably from the numerical results.

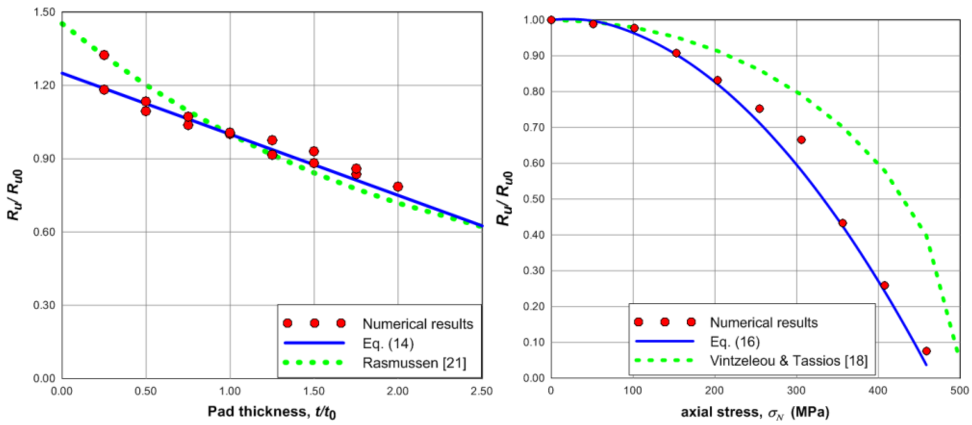


Fig. 9. (a) Normalised shear resistance versus the normalised pad thickness; (b) Normalised shear resistance versus the axial stress of the dowels.

6.6. Effect of the initial axial stress in the dowel

Generally, in precast applications, the grout infill is poured in the beam sleeves after accomplishing the mounting of all precast members. However, many parts of the roof or the floor on top of the connection and the live loads (with less effect due to lower values compared to deal loads) are applied after the hardening of the grout. Therefore, significant axial stresses might develop in the dowels, due to the deformability of the elastomeric pads used. In turn, such axial forces may affect the shear capacity of the pinned connection.

In practice, if significant axial forces are expected to be induced to the dowels due to the aforementioned reason, the reduction factor $\sqrt{1-\alpha^2}$ is used for the shear resistance, with $\alpha = \sigma/f_{yk}$, σ being the axial stress of the dowels (Vintzeleou and Tassios [19]). This factor was derived from tests on double-sided dowels embedded in concrete blocks being practically in contact (without the presence of the elastomeric pad) and for tensile axial load at the dowels applied simultaneously with the shear loading. In the case of precast beam-to-column connections, the axial stresses in the dowels develop at an earlier phase than the shear loading.

In the numerical investigation, the effect of the axial stress of the dowels on the shear resistance R_u of the connection was examined for ten values of the axial stress σ_N, ranging from 0 to 500 MPa. The analyses were performed for connections made of 2Ø25 dowels with cover d = 0.10 m and yield stress f_{sy} = 500 MPa.

The reduction in the shear strength of the connection with the axial stress of the dowels is presented in Fig. 9b, in which the ratio R_u/R_{u0} is plotted versus the axial stress σ_N of the dowels, where R_{u0} is the shear resistance without any axial stress in the dowels.

Based on the obtained results, it is suggested that a correction coefficient $\alpha_{\sigma N}$ should be used for the calculation of R_u if axial stresses pre-exist in the dowels, i.e.

$$R_u = a_{\sigma N} \cdot R_{u0} \qquad (15)$$

Performing a regression analysis on the numerical results, the following equation was derived for the calculation of the coefficient $\alpha_{\sigma N}$:

$$a_{\sigma N} = -4 \cdot 10^{-6} \cdot \sigma_N^2 + 3 \cdot 10^{-5} \cdot \sigma_N + 1 \tag{16}$$

σ_N is in MPa.

It is evident from Fig. 9b that large axial stresses in the dowels lead to significant reduction in the shear resistance of the connection. It is recommended, therefore, to pour the grout infill in the beam sleeves after mounting the roof or the floor elements in order to reduce these stresses.

6.7. Effect of the joint rotation

The experiments performed within the SAFECAST project and the above-presented analyses concern the behaviour of pinned connections under purely shear loading, since no rotation of the beam relative to the column, denoted as joint rotation, was allowed. In real structures, however, in addition to the shear loading of the beam-to-column connections during earthquakes, joint rotations also occur, due to the deformation of the column and the beam. Joint rotations result in additional axial stresses in the dowels, thus, they affect the resistance of the connections.

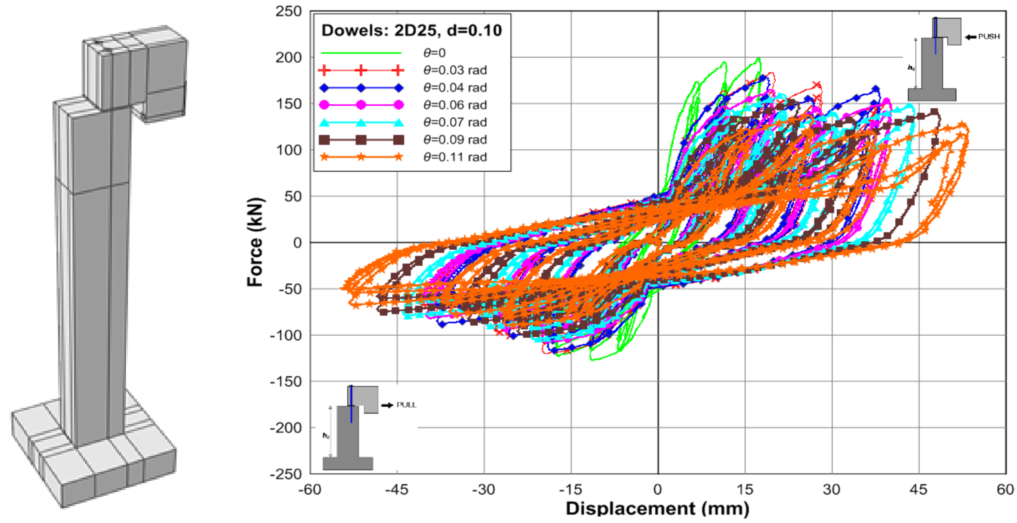
Initially, the applied load F is taken by the shear force V (see Fig. 11a). However, after yield (formation of plastic hinges) and as the displacement and rotation of the joint increases, all the additional load and part of the already applied load is taken by the axial force of the dowel. The plastic moment of the dowel M_P decreases as the axial force N increases due to the shift of the neutral axis. Therefore, the shear force V decreases as the displacement increases. For the ultimate strength (up to the fracture of the dowel), it can be assumed that V is much smaller than N, due to the small value of M_P (the area of the cross section of the dowel under compression will be small for large N, taking under consideration the circular shape of the section). Thus, the strength corresponding to the failure of the connection is practically controlled by the tensile strength of the dowels. However under cyclic loading where the ultimate shear resistance of the connection, R_u, is assigned to the 'yield' strength, the combined failure mechanism of concrete and steel seems to prevail again.

In order to investigate the effect of the joint rotation on the shear resistance of the connection, the numerical model was altered to include the full column (Fig. 10a). In this way, rotations were induced at the joint during the horizontal loading of the beam, caused by the bending deformation of the column. Six column heights were considered, namely 5.0 m; 6.0 m; 7.0 m; 8.0 m; 9.0 m; and 10.0 m. In all cases, the beam-to-column connection was made of 2Ø25 dowels with cover d = 0.10 m and the cross-section of the columns was constant. The thickness of the elastomeric pad, t, varied from 1 to 4 cm. Each model was subjected to cyclic loading, as in the previous analyses. The expected joint rotations for each column height were estimated from a standard seismic analysis (for the same seismic requirements according to EC8).

The numerical results for the model with pad thickness t = 2 cm are shown in Fig. 10b. It is evident that, as the joint rotation increases, the resistance of the connection decreases. Therefore, a reduction factor, a_r, should be introduced in the proposed formula for the calculation of the shear resistance if rotations are present, thus one can write:

$$R_u = a_r \cdot R_{u0} \quad (17)$$

where R_{u0} is the reference resistance for zero joint rotation. The coefficient a_r depends on the expected joint rotation θ, which can be estimated from a standard seismic analysis.



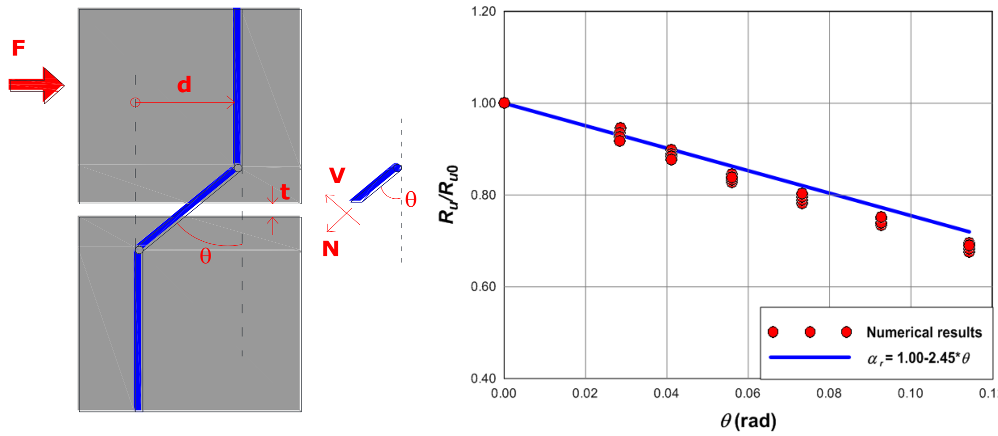
(a) (b)

Fig. 10. (a) Schematic representation of the full column model; (b) Shear resistance of the connection for varying joint rotation for pad thickness t = 2 cm.

For the derivation of the relation between a_r and θ, a regression analysis was performed using the numerical results, which are summarized in Fig. 11b. In this figure, the variation of the ratio R_u/R_{u0}, R_{u0} being the resistance for θ = 0, with the relative beam-column rotation θ is shown. The obtained relation is:

$$\alpha_r = 1.00 - 2.45 \cdot \theta \quad (18)$$

where θ is given in rad.



(a) (b)

Fig. 11. (a) Axial and shear forces developed in the dowel under given combined rotation and displacement; (b) Effect of the joint rotation on the shear resistance.

7. ESTIMATION OF THE SHEAR RESISTANCE

From the numerical investigation presented above, an enhanced formula is proposed for the estimation of the shear resistance of precast pinned beam-to-column connections, to be used in seismic design. The proposed formula considers all the examined parameters and can be written as follows:

$$R_u = C \cdot \alpha_{cov} \cdot \alpha_t \cdot \alpha_{\sigma N} \cdot \alpha_r \cdot n \cdot D^2 \cdot \sqrt{f_{c,\max} \cdot f_{sy}} \qquad (19)$$

where:

R_u is the shear resistance of the connection;

n is the number of the dowels of the connection;

D is the diameter of the dowel(s);

$f_{c,\max}$ is the compressive strength of the concrete or the grout, whichever is larger;

f_{sy} is the yield stress of the steel of the dowels;

α_{cov} is the coefficient related to the concrete cover, calculated by Eq. (12);

a_t is the coefficient related to the thickness of the bearing pad, calculated by Eq. (14);

$a_{\sigma N}$ is the coefficient related to the axial stresses of the dowels, calculated by Eq. (16);

a_r is the coefficient related to the rotation of the joint, calculated by Eq. (18);

C is a general coefficient accounting for: (i) various parameters involved in the shear resistance of the connection, not explicitly considered in Eq. (19), e.g. the value $\pi/4$ related to the area of the cross section of the dowels; (ii) several phenomena, not explicitly considered in the analyses, as the increased local bearing capacity of the concrete in front of the dowels [20], etc.

For the estimation of the most appropriate value of the coefficient C, Eq. (19) was applied to all cases examined within the SAFECAST project and the numerical results were compared with the experimental data. This comparison is presented in Fig. 12a and shows that best fitting of the experimental results is obtained for C = 1.10.

For design purposes, the following alterations to Eq. (19) should be made:

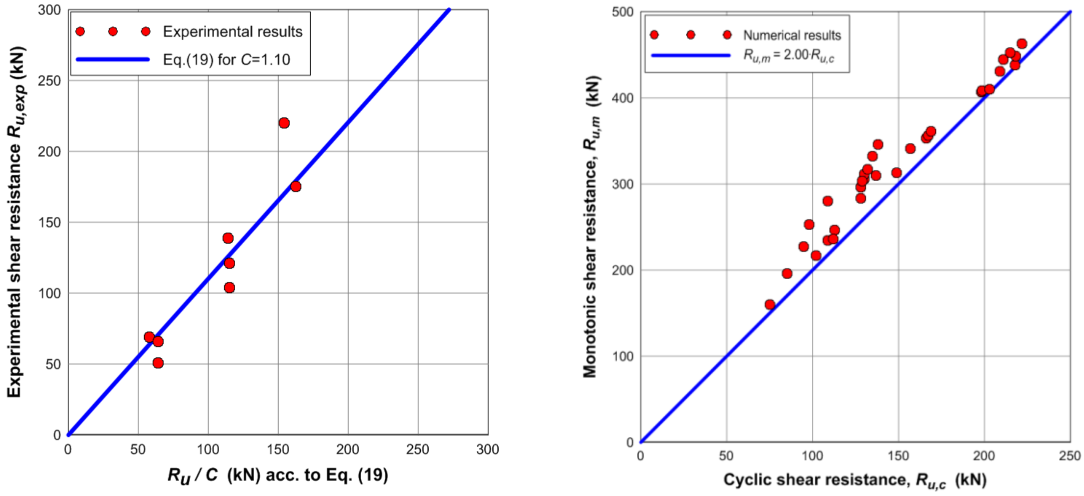
- The design values of the strength of the concrete/grout, $f_{cd} = f_{ck}/\gamma_c$, and the steel, $f_{yd} = f_{yk}/\gamma_s$, should be used in place of f_c and f_{sy}, where γ_c and γ_s are the corresponding material safety factors, typically taken equal to $\gamma_c = 1.50$ and $\gamma_s = 1.15$ [28], and f_{ck} and f_{yk} are the characteristic compressive strength of concrete/grout and the characteristic yield stress of the steel of the dowels, respectively.
- A general safety factor γ_R should be applied to account for several uncertainties, construction deficiencies, etc. It is suggested that $\gamma_R \approx 1.30$, as recommended by the *fib* Bulletin 43 [1].

Therefore, for the calculation of the design value of the shear resistance, $R_{u,d}$, the following formula should be used:

$$R_{u,d} = (1.10/\gamma_R) \cdot \alpha_{cov} \cdot \alpha_t \cdot \alpha_{\sigma N} \cdot \alpha_r \cdot n \cdot D^2 \cdot \sqrt{f_{cd,\max} \cdot f_{yd}} \qquad (20)$$

It should be emphasised that Eqs. (19) and (20) are valid only in case that adequate confining reinforcement exists around the dowels and especially close to the joint interface. Adequate reinforcement (in the case of small covers) around the dowels should mean reinforcement configuration and quantity: (a) capable of undertaking the expected shear force that could be applied to the connection without yielding; (b) capable to ensure compressive strength of the concrete under triaxial stress conditions equal to 5 times the uniaxial compressive strength (assumed value according to Vintzeleou and Tassios [19]).

Also, the dowels should be adequately anchored in the concrete mass at the column side, with proper reinforcement placed around them to confine the concrete and anchor them against pull-out. It is suggested that the dowels should be bolted on top.



(a) (b)

Fig. 12. (a) Comparison of Eq. (19) with the experimental data for cyclic loading; (b) Monotonic vs. cyclic shear resistance according to the numerical results.

8. MONOTONIC VS. CYCLIC LOADING

The proposed formula for the calculation of the shear resistance of pinned connections was based on the numerical results obtained for cyclic response. The resistance under monotonic loading is much larger. According to Vintzeleou & Tassios [19], the shear resistance of a dowel imposed to cyclic loading is equal to one half of the shear resistance of the dowel under monotonic loading. In the experimental investigation performed within the SAFECAST project it was found that the shear resistance of pinned connections under monotonic loading, $R_{u,m}$, is generally larger than twice the one under cyclic loading, $R_{u,c}$.

In Fig. 12b, the comparison of $R_{u,m}$ (monotonic resistance) with $R_{u,c}$ (cyclic resistance) according to the results of the numerical analyses is presented. As mentioned above, the shear resistance of the connection under cyclic loading was calculated from the ultimate resistance observed in the pull direction and was set equal to the yield force of the corresponding bilinear representation of the force-displacement curve. The shear resistance under monotonic loading was set equal to the maximum force attained before the failure of the dowels for loading in the pull direction. Fig. 12b shows that the above-mentioned relationship: $R_{u,m} = 2R_{u,c}$ is a rather conservative assumption, since all the experimental data lie above this line.

9. PRELIMINARY FINDINGS ON THE OVERALL RESPONSE OF THE CONNECTION

The aim of the present research is the prediction of the shear resistance, R_u, of a precast pinned beam-to-column connection under cyclic shear loading; however the overall force-displacement envelope curve of this type of connections is of particular interest. In the literature, relevant information is given in this direction by Vintzeleou and Tassios, reflecting the relationship between shear force and shear displacement of a one-sided dowel, failing by combined steel/concrete failure, as schematically presented in Fig. 13. Note that the displacement in Fig. 13 refers to the slip of a one-sided dowel. In case of double-sided dowels with symmetric conditions the figure gives half the shear displacement of a dowel connection.

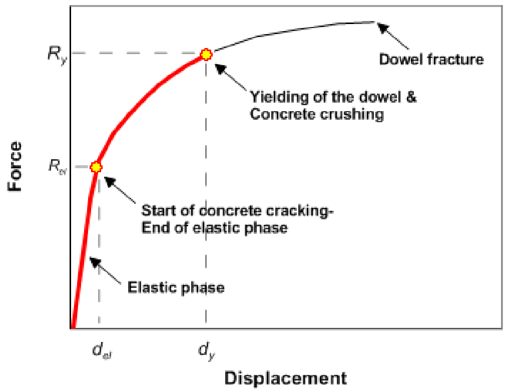


Fig. 13. Force-displacement envelope for a one-sided dowel under monotonic loading by Vintzeleou and Tassios.

It is in the future plans of the authors to extend their research and provide information about the overall force-displacement response of the connections under investigation. Their preliminary findings, up to now, are directed in proposing an idealized elastic-perfectly plastic bilinear force-displacement diagram to be used for design purposes (as in Fig.14).

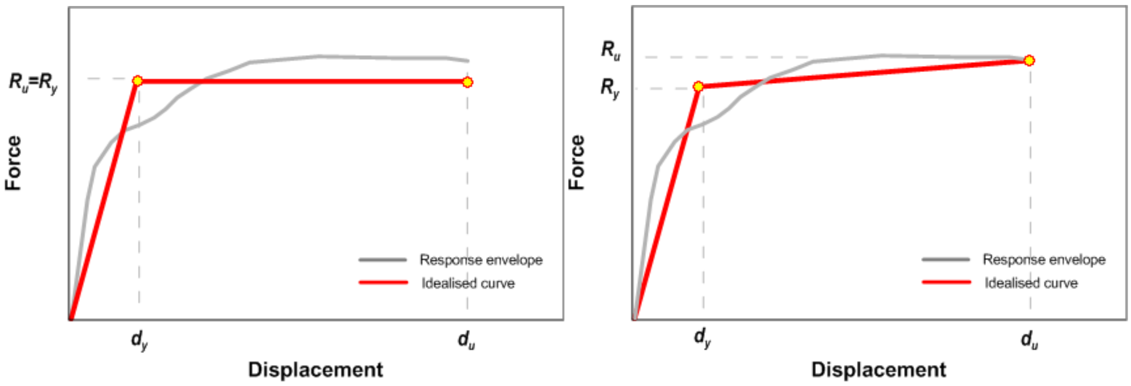


Fig. 14. Idealised bilinear force-displacement diagram for precast pinned beam-to-column connections under: (a) monotonic loading; (b) cyclic loading.

Under monotonic loading, the ultimate resistance of the connection could correspond to the value corresponding to the fracture of the dowels (Fig. 14a). Under cyclic loading a precast pinned beam-to-column connection should be designed according to Eurocode 8 as an overdesigned connection. Therefore the connections are expected to behave elastically during earthquakes while plastic hinges are expected in other parts of the structure (i.e. columns). The connection should be designed taking into account the values of R_y and d_y as given in Fig. 14b. However the connection if it is stressed beyond its elastic limit due to unexpected reasons, it can bear significant post-yielding displacements before it loses its strength, showing shear ductility capacity larger than 4.0.

Vintzeleou and Tassios defined the values of R_{el}, d_{el}, R_y and d_y for one and double-sided dowels under monotonic loading and suggested a relation between monotonic and cyclic shear resistance equal to 50%. The value of d_y proposed by Vintzeleou and Tassios seems to be confirmed by the present preliminary results. However the authors, for informative purposes at the moment, could propose the following relationships (Eqs. 21-23):

$$R_{y,m} = 0.50\, R_{u,m} \tag{21}$$

$$R_{u,m} = 2R_{u,c} \tag{22}$$

$$d_{y,\,c} \approx 0.20\, d_{u,c} \tag{23}$$

10. CONCLUSIONS

A numerical parametric investigation was performed and a new formula is proposed for the shear resistance of precast RC pinned beam-to-column connections under cyclic loading. The proposed formula addresses the case where the failure of the connection occurs with simultaneous flexural failure of the dowel and compression failure of the concrete around the dowel, expected to occur either when (a) adequate concrete cover of the dowels is provided (d > 6 D) or (b) adequate confining reinforcement (as defined previously in Section 3 is foreseen around the dowels in the case of small concrete covers (d < 6 D). The presence of confining reinforcement around the dowels in case of small concrete covers results in the change of type of failure from brittle to ductile. For any other type of failure the formula cannot be reliably used to estimate the strength of the connection, since the mechanisms of response are completely different.

The parameters examined herewith are: the number, n, and diameter, D, of the dowels; the strength of the materials (concrete, grout, steel); the concrete cover of the dowels in the loading (denoted by d) and the normal to the loading direction (denoted by d_n); the thickness, t, of the elastomeric pad; the pre-existing axial stress in the dowels, σ_N; the beam-column relative rotation (joint rotation); and the type of loading (monotonic or cyclic). Some of these parameters, such as the number and diameter of the dowels and the strength of the materials, are examined also by other researchers and were once again confirmed herewith.

The conclusions of this paper can be then summarised in primary contribution order as follows:

- With regard to the concrete cover, d, of the dowels in the loading direction, the shear resistance decreases for d/D < 9. The drop in the resistance is about 10% for d/D = 6, but it reaches 35% for d/D = 4. Values of d/D smaller than 4 are not recommended and are not addressed by this formula. The concrete cover of the dowels in the normal to the loading direction, d_n, does not seem to affect the resistance considerably, provided that it is at least equal to 4D.
- In precast technology the use of elastomeric pads for the seating of the beams is common. The analyses showed that the ultimate shear resistance of the connection decreases as the pad thickness increases, while the ultimate displacement increases.
- Existing axial stresses in the dowels due to the loads of the roof/floor result in lower shear resistance of the connections.
- The development of relative beam-column rotation during earthquakes also leads to a reduction of the shear resistance of the connections.

- With regard to the number and diameter of dowels, it was confirmed that the shear resistance is proportional to the product ($n\ D^2$), as suggested by previously proposed formulae.
- The strength of the materials (grout/concrete, steel) affects the shear resistance proportionally to their square root, as also suggested by existing formulae. It is noted, however, that, if the strength of the grout is different than the strength of the concrete, the resistance of the connections depends on the larger strength.
- The shear resistance of pinned connections under monotonic loading is generally larger than twice the one under cyclic loading.

ACKNOWLEDGEMENTS

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